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EVALUATION OF SUPPORT SYSTEMS: ROCK BOLTING

- FIRST INTERIM REPORT

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EVALUATION OF SUPPORT SYSTEMS: ROCK BOLTING - FIRST INTERIM REPORT

by

M. Gyenge*

SUMMARY

The objective during the present phase of the Material Properties and Support Systems Project is to evaluate the effectiveness and limitation of the various underground support systems.

The study, completed during the first year of the project, identified those factors which generally have major influence on the stability of underground openings and consequently on the support design. Rock bolting practice used in the Ontario mines and the methods of rock bolting designs were reviewed. An evaluation of various testing techniques and testing equipments, used in establishing the strength and deformation properties of rock up to and beyond fracture, was also completed.

Based on the results of the completed study, the areas of laboratory and field investigations were identified, the program relating to the development of post-failure rock testing procedure was established, and the steps of the analytical work required to produce the draft guidelines on rock bolting were outlined.

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INTRODUCTION

Ground support systems vary with mining method, excavation technique and ground conditions encountered. The requirements for ground support and the effectiveness of presently used support systems at depths below 1000 m are not well known.

The objective of the Material Properties and Support Systems Project during the present phase is to evaluate the effectiveness and limitations of the various underground support systems. In a subsequent phase the results of this and other concurrent investigations are to be integrated within ground control guidelines for deep hard rock mining.

The evaluation of each support system is to be based on a series of systematic studies. Within these studies laboratory and field investigations are to be conducted, as necessary to define the requirements, specifications, techniques and procedures relating to the laboratory and field testing, as well as to the design, the installation and the construction of the specific support system.

These and the other studies of concurrent projects are to be completed within an estimated five year period. To integrate the results and to publish it in the form of guidelines could take an additional two or three years. In order to transfer the information as it becomes available, the results of the completed investigations are to be summarized in the form of individual technical reports for immediate use by the industry.

The objectives of the investigations during the first year by the Material Properties and Support Systems Project were the following: to study the presently used methods of designing underground support systems in order to define those factors which generally have major influence on the stability of underground openings and consequently on the support design; to study the present practices of rock bolting design, installation and testing in order to define the research and analytical work required for the development of rock bolting guidelines; and to study and evaluate various testing techniques and testing equipment used in establishing the strength and deformation behaviour of rock up to and beyond fracture in order to develop the testing procedure to be used in laboratory post-failure strength determination.

CANADIAN UNDERGROUND BASE METAL MINING

MINING METHODS

In underground mines the ores are won by stoping. Stoping is a general term applied to cover any mining method. The percentage of the tonnage of Canadian base metals mined by various underground mining methods, after Hedley (1), is given in Fig. 1. Accordingly, cut and fill methods are predominant, representing almost 50% of underground production. It is followed by open stoping and sublevel caving with 28.9% and 16.2%, respectively. Room and pillar and shrinkage stoping methods respectively represent 3% and 1.7% of underground production. The remaining 2.4% is mined by various other methods.

Somewhat similar information is given in Fig. 2, which shows the number of mines using these mining methods. Figure 2(a) represents the entire Canadian underground base metal mining, based on the survey of 56 mines (2). Figure 2(b), based on a survey of 40 mines, reflects the underground base metal mining practice in Ontario (3). From Figs. 2(a) and 2(b) it is evident that while open stoping is the most commonly used method on a Canada wide basis, the most widely used method in Ontario is cut and fill. However, if production figures are also considered, as represented by Fig. 1, the survey results of the Ontario mines can be considered as more representative of Canadian underground base metal mining practices.

A more detailed break-down of mining methods used in the Ontario mines is given in Fig. 3. A few of the 40 mines surveyed use only one mining method. The majority of mines use two or three methods. The number of mining methods employed reaches five in some cases. The choice of the mining method is influenced by several factors. The type and shape of the orebody is one of the primary factors. The strength of the ore and of the walls is the other primary factor. Safety and economy are equally important factors in the selection of a mining method. In general the larger the opening that can be left unsupported, the greater the degree of mechanisation that can be employed, and as a result the greater the safety and economy of the operation.

MINING SUPPORTS

The survey of the forty Ontario mines provides information on the artificial support systems used. Graphical presentation of the survey result with regard to support systems is given in Fig. 4.

Figure 4 reveals that rock bolting is used in all 40 mines. Both hydraulic fill and timber support is used in about 50% of the surveyed mines. Shotcrete and gunite is used by 14 and eight mines, repectively. A steel set support system is used only in one mine.

It should be noted that Fig. 4 includes only artificial support systems. Pillars, whether they are rib, sill or crown pillars, represent natural mine support. Consequently, they are excluded from Fig. 4.

DESIGN OF UNDERGROUND SUPPORT SYSTEMS

The fundamental principle of mine support design is that the major structural support element is the rock itself. In some cases the stability of underground opening is achieved entirely through the use of the self-supporting capability of the rock material. In other cases additional structural materials, such as steel, concrete, wood, etc., must be used as reinforcement.

The design of a support system requires a knowledge of active mine loads and the strength and deformation properties of the structural elements which constitute the support system. Rock mass discontinuities cause considerable variation in the strength and deformation properties of rock mass which can effect load distribution around mine openings.

The various factors which affect the stability of an unsupported opening in rock are the following:

(a) rock type;
(b) geologic structure;
(c) natural in situ stresses;
(d) size and geometry of opening;
(e) strength and deformation properties of rock;
(f) environmental factors; and
(g) excavation method.

All of these factors are interconnected. The stability analyses of an unsupported opening and the design of an appropriate support system will have to consider all the factors simultaneously.

A brief description of these factors as well as their interplay with others is given below.

ROCK TYPE

In designing any underground support system the first information asked is the rock type. Rock type is associated, though practical experiences, to the overall behaviour of rock mass around openings.

Igneous rocks are generally massive. The uniaxial compressive strengths of intact igneous rock samples are high. Therefore, unsupported openings in these rocks can be stable to depths greater than 2000 m, unless the igneous rock mass is extensively jointed or fragmented by faults and shear zones. Rock spalling may occur in areas of high ground stress.

In sedimentary rocks the stability of unsupported openings varies widely. Some sandstones and shale formations have sufficiently high strength that wall rocks are self-supporting at considerable depth. Openings in other sandstones and shales must be supported even at shallow depths because of their low strengths and other undesirable properties. For example, shales may have swelling or air-slaking characteristics.

In massive unbroken limestone and dolomite the unsupported underground openings may be stable to depths of 1500 m but to a much lesser depth where the rock is porous and of lower strength. However, limestone and dolomites, as well as gypsum and rock salt, often contain sinkholes, solution caverns or enlarged fissures by solution that will cause serious stability problems.

Anisotropy and bedding are characteristic features of sedimentary rocks. Due to these factors the rocks behave differently in different directions. If for example, the unsupported opening is in a horizontally bedded formation, the roof may tend to separate along the bedding planes and to fall out when the developed beam fails. On the otherhand, if the lamination is vertical and perpendicular to the axis of the opening, the roof would tend to be stable.

The intact strength of metamorphic rocks, such as quartzite, gneiss and marble are usually high. Therefore, the unsupported openings in massive deposits of these rocks could be stable virtually at any depth. However, some metamorphic rocks such as schists and some gneisses, are strongly anisotropic. The orientation of the anistropy with respect to the opening must be considered. The quartzite rocks have high strength, but because they are brittle, spalling or even rock bursts could be a problem in zones of high stress.

GEOLOGIC STRUCTURE

Structural geologic features such as joints, foliations, shear zones and fault zones significantly influence the stability of unsupported openings. Joints and foliations are by far the most common. They might or might not represent stability problems. However, shear zones and fault zones although being less prevalent, are always the subject of great concern. Openings within sheared, fractured and crushed rock, frequently with soil-like characteristics, normally require additional support.

Every rock mass possesses joints to some extent. In general, there are two or three prominent joint sets and possible several minor sets. In metamorphic rocks one prominent joint set is usually parallel or subparallel to the foliation surfaces; in sedimentary rocks one set of joints usually parallels the bedding.

To analyse the influence of joints on the stability of openings, the following factors must be considered: (a) the orientation of the joint surfaces, i.e., angles relative to the roof and walls of the openings, (b) the spacing or frequency of joints, (c) the characteristics of the joint surfaces, i.e., tightness, roughness, alteration, and (d) the characteristics of the gouge material within the joints, i.e., strength, deformation and physical properties. Each of these factors for a given opening may assume overriding importance either alone or in combination with others.

The flat-lying joints (0° - 30°) in combination with nearly vertical cross-joints could be the source of instability of the roof. The steeply dipping joints (45° - 90°) which are parallel or subparallel to the walls could cause stability problems with the walls.

The relative stability of an opening, with respect to joint spacing, depends on the ratio between the joint spacing and the dimensions of the opening. Small openings in a blocky rock mass, i.e., large joint spacing, are stable, while large openings in a jointed rock mass with small spacing are usually unstable.

The characteristics of the joint surfaces and of the gouge material within the joints effect the stability because of their important influence on the strength and deformation properties of the jointed rock mass. In general, openings excavated in a formation with tight joints and with rough joint surfaces are more stable than those in a jointed rock mass where the joints with smooth surfaces are open. Joints, healed through secondary mineraliza-

tion such as quartz will lessen the concern about stability, while joints with gouge materials such as clay require the utmost attention.

NATURAL IN SITU STRESS

The stress distribution around an opening is the result of the readjustment of the natural in situ stresses disturbed by the excavation. Consequently, the distribution and the magnitude of resultant stresses will depend, in part, on the distribution and magnitude of natural in situ stresses that prevailed prior to the creation of the opening.

The results of in situ stress measurements, conducted in the Canadian Shield, indicates considerable variations with site. This is to be expected, as the geological setting, the structure and the elastic properties of rocks vary significantly between locations. In general, the minimum principal compressive stress is oriented vertically, and in most cases can be assumed equal to the weight of the overburden at that point. The maximum compressive stress directions varies, but a sub-horizontal easterly direction is favoured. The ratio of the average horizontal stress to vertical stress approaches unity near 3000 m depth. In situ stress measurements obtained close to tectonically disturbed areas differ considerably from these average results.

The components of the resultant stresses, which are functions of the natural in situ stresses, influence the stability of an unsupported opening in various ways. The radial stresses cause inward movement of the periphery of the opening. The stability of the opening will depend on the geologic structure and the strength and deformation properties of the rock mass, as well as, on the size and geometry of the opening.

The effect of tangential stress component on opening stability is equally influenced by these factors. In the case of a large-span opening, the resultant tangential stress in the mid-section of the roof can be tensile. This could result in the opening of joints and consequently rock falls in the roof. The prevailing tangential stresses around the openings are, however, compressive. Providing that the ratio between the tangential stress and the uniaxial compressive strength of rock substance is less than one, the tangential stresses could improve the stability of the underground opening. This is because the joints became tighter and because the shear resistance of joint surfaces increases, due to the compressive forces. However, if the ratio between the tangential stress and the uniaxial compressive strength of rock substance approaches or exeeds one, failure of the surface rock will occur.

Discontinuities with unfavourable orientation or with unfavorable properties, besides contributing to failure, can also influence the redistribution of the natural in situ stress field. Mineralized veins with extremely high elastic constant values have low deformability. Soil-like gouge materials are just the opposite, possessing high deformability and low elastic constants. Materials with high elastic constants attract, while materials with low elastic constants refuse additional loads. Such discontinuities can cause stress perturbation around an opening which contributes to instability.

SIZE AND GEOMETRY OF OPENING

The stability of an unsupported opening depends on the size and geometry of the opening in relation to the properties of the rock mass.

A uniform rock mass can be regarded as an elastic continuum which behaves essentially elastically under the imposed loads. If this assumption is justifiable then the elastic solutions can be applied to obtain a first estimation of the stress distribution around the opening.

If the stresses around the opening, as calculated by the elastic solutions, are within the elastic range of the intact rock, then the opening is theoretically stable. This applies to all sizes of opening, because the elastic solutions are independent of the size and vary only with the geometry of the opening. Therefore, it is desirable to select the geometry that results in the least severe stress conditions at the periphery of the opening. However, because the most important factor in selecting geometry is the final function of the opening, the possibility to modify a geometry is limited.

The elastic stability analysis of unsupported openings in an elastic rock continuum can be performed by considering the ratio of the maximum theoretical tangential stress at the periphery of the opening and the uniaxial compressive strength of the rock. If this ratio is smaller than one the opening is theoretically stable, independently of the size of the opening. When the tangential stress approaches the uniaxial compressive strength, the failure of the wall material is initiated.

If the rock behaves as an ideal elastic material, the mode of failure is shear failure. However, the behavior of some rocks may not be ideal and therefore they may fail although the ratio between the tangential stress and the uniaxial compressive strength is smaller than one. The characteristic failure mode is referred to as spalling or popping. Spalling is the result

of tensile stresses induced by the Poisson effect on the tangential compressive stresses.

When the above stress/strength ratio is larger than one, a platic zone is theoretically formed around the opening. In this case, the elastic-plastic solution may be applicable to obtain a first estimation of the stress distribution around the opening. As a result of the formation of a plastic zone of material around the opening, the unsupported openings could theoretically become stable. Some support would likely be necessary, however, to retain the failed rock at the periphery of the opening.

In order to use the elastic-plastic solution the extent of the plastic zone must be known. This depends on the strength parameters, i.e., coefficient of friction and cohesion, of the rock. Assignment of realistic values to these parameters is always a difficult task.

The stability of an unsupported opening in discontinuous rock cannot be analysed by theoretical equations. Although the stability is influenced by the intact strength properties of the rock, the orientation, frequency, type and condition of the discontinuities in the rock mass are the controlling factors. Sliding, rotating and loosening of rock blocks are the causes of the instability in a discontinuous rock mass.

Block movements are the result of the re-adjustment of the natural in situ stress field following the creation of the opening. Block movements continue until a condition of equilibrium between the driving forces and resisting forces, along the discontinuity surfaces, is reached.

Under a favorable combination of geometry, orientation and rock quality the surface rock blocks form a natural arch. In this case the unsupported opening could be stable. Some blocks with marginal equilibrium might require support even under this favorable condition. Under an unfavorable situation, equilibrium conditions can only be established by use of an appropriately designed support system. In general, discontinuous rocks masses must be considered as potentially unstable.

Unless there are specific structural features, openings with rounded or curved surfaces in cross sections are more stable than openings with flat straight line sections. From a stability point of view, therefore, circular and eliptical shapes are preferred to square or rectangular shapes. Due to the surrounding stresses, there is a general inherent trend within the rock mass towards self-shaping to the more stable configuration.

split tube is forced into a 1 3/8 in. diameter drillhole, due to the spring action of the compressed tube, a radial force develops along the walls of the hole. This force generates a frictional resistance to sliding of rock on the steel. The friction increases as the outside surface of the tube rusts.

It cannot be grouted to protect against excessive rusting, which can be a problem where long term support is required. The drillhole diameter is critical for the support effectiveness of the system.

Split sets are used in the mining industry where relatively short term light support is required. As indicated in Fig. 5, split set bolting is used by nine out of 40 Ontario mines.

DESIGN OF ROCK BOLTING

The use of rock bolts as the essential support system, alone or in combination with other structural material, must be examined in conjunction with the material to be bolted. The major factors which determine the use of bolts are:

(a) the type of strata around the opening;
(b) the method of working;
(c) the stress developed around the opening.

These factors are interrelated. The interplay of these factors has already been discussed under the heading of "Design of underground support systems", where the procedures of support design, in general terms, are also given.

The same design steps are applicable in rock bolt design. However, one must remember that rock bolting does not really support an opening. The overall opening must be basically self-supporting with firm walls, abutments or pillars. The principle function of rock bolting is to reinforce or stabilize the rock at the surface of the opening.

The objective of rock bolting design is to specify the bolting system in terms of bolt type, diameter and length of bolt, bolt spacing, installation and inspection procedures, monitoring and testing procedures.

STRENGTH AND DEFORMATION PROPERTIES OF ROCK

The acting force on the rock surrounding an opening is in the form of a distributed load, referred to as the resultant stress field. The mode of the distribution of the stresses will depend on the following factors: size of opening, geometry and shape of opening, geologic structure of the rock mass, and natural in situ rock mass properties.

The stability of the unsupported opening will depend on those strength and deformation properties of the rock mass which are attained following the excavation. The attained or resultant properties will depend on the excavation method and on environmental factors.

If the underground opening is small in comparison to the spacing of discontinuities, i.e., slightly jointed massive rock, the assumption can be made that the rock mass is a continuum. The resultant stress distribution in this case can be established by elastic solution and the stability of the unsupported opening will be controlled by the uniaxial compressive strength of the intact rock.

In analyzing the stability of an opening, the tangential stress components at critical points of the periphery are compared with the compressive strength of the intact rock. If the acting stess is lower than the compressive strength, then the opening is stable. Rock failure at the periphery will occur when the acting stress approaches the value of the uniaxial compressive strength. The mode of failure is either shear or spalling failure.

When the acting stress exceeds the uniaxial compressive strength, a plastic zone is theoretically formed around the opening. The stability of the rock within the plastic zone will be governed by the strength parameters, coefficient of friction and cohesion, of the rock. These parameters depend on the confinement of the rock. Because the confinement along any wall section varies with depth, in accordance with the prevailing stress condition, it is a difficult task to assign realistic values to the strength parameters required for stability analyses.

The instability of openings in a discontinuous rock mass is caused by the loosening, sliding and rotating of rock blocks. Following the excavation of the opening, as a result of re-adjustment of the natural in situ stresses, the movement of rock blocks continues until an equilibrium between the driving and resisting forces along discontinuity surfaces is reached.

Theories of Bolt Support Mechanisms

Considerable research efforts have been devoted in the past to define the underlying support mechanism of rock bolting with the aim of establishing a standard method of rock bolting design. Investigations have been based on several assumed modes of reinforcing action by rock bolts. The obtained results reveal that each mode explains only certain factors of rock bolt behaviour, but does not provide overall design criteria. The generally accepted conclusion is that the behaviour of the bolts is a combination of factors contained in the following theories of bolt support mechanisms.

The laminated beam theory may provide a close simulation of reinforcing mechanism in the case of bedded rocks. Theoretically a self-supporting laminated beam can be formed if two or more thin to medium thick bedded strata are bolted together. The results of theoretical and model studies on laminated beam theory were summarized by Obert and Duvall in a design chart (4). The chart is based on continuous strata, including a strong stratum which will stay intact and be able to bridge over firm abutments. More than a few joints in a bed and slabbing of the abutments invalidate the chart.

The second theory states that rock bolting can only be effective if the rock bolts are anchored beyond the probable failure zone around the opening. Roof falls in stopes, roadways and tunnels are usually terminated in a natural arch shape or at a strong overlying stratum. The limit of the probable failure zone can be estimated by calculation or determined by field measurements. The loose rock within the failure zone is stabilized by the support system consisting of rock bolts, anchored within the rock mass beyond the failure zone, with large bearing plates and wire mesh held by the plates.

The third and possible the widest held theory is that the tensioned rock bolt has the effect of clamping and compressing the rock fragments within a cylindrical zone around the bolt. If the bolts are spaced sufficiently close to eachother then these compressed cylindrical zones will form the equivalent of a masonary bridge and the fragmented rock mass becomes self-supporting. Considerable research was conducted with respect to this theory by both the Australian Snowy Mountains Hydroelectric Authority and CANMET (5,6). These investigations resulted in codes of practice for rock bolting, frequently used by the mining industry.

The fourth theoretical explanation of the support mechanism operative with rock bolts concerns surface effects. The rock bolts are assumed to pre-

The stability of the rock blocks will depend on their size and orientation in relation to the opening and on the strength parameters of the discontinuity surfaces. These strength parameters, i.e., cohesion and friction angle, will depend on the type of the material along the contact surfaces between the blocks and on the condition of contact surfaces. Smooth surfaces filled with clay-like gouge material, saturated, which has zero cohesion and very low friction angle, are the worst cases in terms of stability. Clean and rough joints represent the upper end of the stability scale. Movement along the latter type of joint is resisted by high surface values of cohesion and friction. This is especially so for small initial movements, when the strength parameters are at their peak. However, if the movement along such joints is continued, resistance will be reduced because the residual values of the strength parameters will be reached.

ENVIRONMENTAL FACTORS

Environmental factors which may influence the stability of an unsupported opening are: neighboring excavation, nearby blasting and previous mining.

The neighboring excavation, i.e., a parallel opening excavated on the same or close to same level, influences the stability of an opening in two ways. One influence is related to the ground stresses. Ground stresses change significantly when an excavation is made. Therefore, a nearby second opening is excavated within a stress field that was changed from the in situ state by the stress redistribution that occurred when the first opening was excavated; similarly the stresses around the first opening are changed by the redistribution caused by the second opening.

The second influence is related to rock mass properties. The strength and deformation properties of a rock mass around an opening are changed by stress and strain changes and by the loosening and movements caused by excavation. As with the stresses, there is an interplay between these property changes. Each opening is affected by the rock mass property changes it produced, as well as, by those produced by openings excavated either before or after it.

In the case of massive and slightly jointed rock mass, the effect of the neighboring excavation can be estimated with acceptable accuracy, by superpositioning the effects computed by elastic solution. Simple superposi-

tion is not as satisfactory in case of elastic-plastic materials. The reason is that the material is stress-path dependent and also that the position of the elastic-plastic boundary changes with a change in stress level. Consequently, each effect has to be evaluated separately.

In a typical disontinuous rock mass the actual changes in stress distribution and the changes in material properties cannot be based on either theoretical or experimental results. Changes in stress distribution cannot be avoided. In general, however, changes in material properties are somewhat controllable and can be minimized by appropriate excavation techniques. Changes nearly always lead to a decreased rock mass strength. As consequence the results are: a decrease in the stability of the opening and an increase in the support requirements.

With respect to nearby blasting, the possibility of damage to the opening due to ground vibrations must be considered. The particle velocity criterion for damage can be applied to establish the blasting details to control blast damage.

The effect of previous mining depends on the relative position of the old and new excavations. If the new opening is located above a previously mined area, the excavation will take place within the subsidence zone where the ground may be more broken and fractured than that of the unaffected ground. Furthermore, depending on the location within the subsidence zone, the rock may be subjected to tensile or increased compressive stresses, that are quite different from those encountered in an undisturbed region. The combination of broken rock and high compressive stresses may produce unexpected rock falls, shear failures or popping rock.

If the new opening is located beneath a previously mined area, the opening will be excavated within a rock mass with redistributed stress field and with properties influenced by the previous excavation. The vertical component of the prevailing stress field may be smaller or larger than that of the unaffected ground, depending on the horizontal and vertical distances from the mined area and on the method of the mining. The difference between the horizontal components of the redistributed and of the natural stress fields is, in general, less significant. The change in strength and deformation properties of the rock mass below a mined-out area is expected to be relatively small.

EXCAVATION METHOD

The excavation method used can have significant influence on the stability of a unsupported opening. The uniaxial compressive strength of nearly all types of rock is high enough that unsupported small openings will be stable when the rock behaves as a continuum, i.e., when the rock is massive and only slightly jointed. However, a discordant method of excavation could cause loosening and fracturing of the wall rocks. This disturbance could change the material behavior in a zone near the opening from that of a continuum to that of a discontinuous material.

In an already discontinuous rock mass the excavation related disturbance will increase the depth of loosened zone. It will tend to extend out, until a stable condition is reached. The consequence of excavation disturbance is a larger volume of loosened rock, which will have to be supported.

The depth of the disturbance depends on the excavation method used. There is no theoretical method available to predict the disturbed depth. The available data, based on experiences, seems to indicate that for an average size drift the disturbance is at least one meter. This will increase as the dimensions of the opening is increased.

This average loosening depth applies when general blasting practice is used, with average spacing, burden and size of charge. Over-charge will increase the depth of the disturbed zone. Improved blasting techniques can, however, significantly reduce the depth of the loosened zone. Due to the large number of the involved factors, however, there is no sufficiently accurate information available to form a basis for relating the blasting technique and the loosened zone depth.

The stability of a machine-driven opening, e.g., bored raise, is easier to predict. Compared to the fracturing and loosening of the rock by blasting, the machine causes little disturbance. The case of massive and only slightly jointed rock mass, that behaves as an essentially continuous elastic material, and of machine-driven circular opening, represents a classical example for the use of theoretical equations in stability analyses.

PROCEDURES OF SUPPORT DESIGN

The underground openings in mines are excavated for various functions in various sizes, shapes and geometries. They are excavated either to provide access to the mining areas or to provide the working area itself.

Underground openings of the first group will usually have the same life span as the mine. As they have to be maintained in safe condition for a longer time period, the support systems used are generally one of the more permanent types. Safe support of working areas is needed until ore within the reach of each specific area is extracted. Thus their support systems are of less permanent type.

The support system design's role is to determine the most economical support system suited to the specific environment and condition of the opening which enables the opening to fulfill its function with adequate safety. Independently of the support system, design of a support system follows the steps, as briefly described below:

Design Data Collection

The geometry, size and shape of opening, the function of the opening, the excavation method, the environmental factors are all defined by the mining plan. Once the mining method is decided, the mining plan, i.e., mine development, lay-out, mining sequence, can be worked out. Consequently, these data are more or less fixed by the mining method. There are some limited possibilities to modify these data, if the support design suggest to do so. However, major modifications are usually connected to a change in mining method.

The rock types and their distribution are generally known. Geologic maps are prepared during the exploration stage of the mine site. These maps usually give all the information required to make an initial stability assessment of the intended openings, based on the rock types involved.

The usual geologic maps, however, almost never provide the structural information with the details required for the support design. They may provide data on the major geologic features such as faults and shear zones. Critical joint systems with possible stability problems are usually indentifiable only through the completion of a well defined program of structural geology investigation.

The natural in situ stresses are established by various stress measurement techniques. These measurement procedures require highly specialized equipment and skill. If, from previous measurements, the natural in situ stresses for the region are available they can be used as the approximate stress field of the site.

The strength and deformation properties of the rocks are determined through a well defined testing program. This program is tailored to suit the requirements of the specific support system, involving the appropriate type and number of laboratory and field tests. Testing of structural elements of the support system, other than rock, is part of the program.

Stability Analyses of Unsupported Opening

The aim of this step is to determine support requirements. The underlying principle in the analyses is that the major structural element of the support system is the rock mass itself. The self supporting capability of the rock mass for each opening is assessed for the conditions defined by the collected data.

The acting load is represented by the resultant stress field. The stress distribution around the opening and the magnitude of the stress components, as the function of the natural in situ stresses, the geometry and shape of the opening and the deformation properties of the surrounding rock mass, is established by using the most appropriate analytical method. The effects of the other factors influencing the distribution and magnitude of the stresses are also established. The combined stresses represent the load system which is acting upon the surrounding rock mass affected by the opening.

The analytical effort depends on the degree of complexity of the involved factors. In the case of a small opening with simple geometry, excavated in a slightly jointed uniform rock mass, the resultant stresses can be calculated manually. Contrary, for large openings with complex geometry excavated in a rock formation with complex structural features, especially if environmental factors are also involved, the analysis calls for computer aided calculation.

The stability of the opening will depend on whether the surrounding rock mass affected by the opening is able or unable to withstand the acting stresses. The stability of the opening is assessed by comparing the maximum acting stresses with the rock strength. The anticipated failure mechanism, corresponding to a specific loading and structural condition, will determine which strength property is used in the comparison. The uniaxial compressive strength is used in case of massive slightly jointed rocks. The ratio between the acting tangential compressive stress and the uniaxial compressive strength is a measure of the stability at the periphery of the opening. Away from the

surface of the opening the stability of a slightly jointed rock mass, however, is controlled by the triaxial strength parameters. In the case of large openings, in bedded deposits with slight cross-jointing, the tensile strength could be the controlling factor.

As the frequency of jointing increases the stability of the rock blocks will determine the stability of the openings. The sliding of blocks, with critically oriented joints, is controlled by the strength parameters along the joints. The rotation of blocks, however, is controlled by both the crushing strength of rock substance and the strength parameters of the joints. The extreme case of jointing is represented by the highly fractured rock mass, such as shear zones and faults. The rock mass stability in such conditions is governed by the strength parameters of the weakest involved rock material.

The stability analyses of the unsupported openings must also consider the interplay between the acting stresses and strength properties. The strength and deformation properties of rock mass around the openings are changed by the deformation of the opening, i.e., loosening, strain, movement along joint planes caused by the stresses. The changes in rock properties in turn, result in further redistribution of the stresses. This interplay continues until the state of equilibrium is reached. Some of the materials, under continuous load, display creep characteristics, i.e., continuous deformation with time. Strength parameters along joints, due to sustained sliding, will be reduced from peak to residual values. It is possible to relate all these changes to the time elapsed from excavation.

The ultimate aim of the stability assessment of an unsupported opening is to deliniate the rock mass around the opening into volumes with various stability conditions. The assessment should also reflect the time related changes in these conditions.

Support Design

The result of the stability assessment of unsupported opening is used to establish the support requirements. The underlying principle of support design is to reinforce the unstable or potentially unstable volumes of rocks around the opening in order to help the rock material to support itself.

The support system will have to fulfill various criteria. The most important criterion is adequate safety. The other criteria are related to the required permanence and the economics of the support and to the function of the supported opening.

The selected support system, best suited to the above criteria, is designed to reinforce the rock volume which is assumed to be critical from the safety point of view. The structual elements of the support system are dimensioned to the design load which can be obtained from the result of the stability assessment of the unsupported opening.

Some systems must be designed to withstand the total weight of the critical rock volume. The design loads on the others may be only a certain portion of the total weight. It depends on the support mechanism of the system, i.e., the interaction between the supported rock and the support. In establishing the stability criterion for the supported opening the interplay between the strength the deformation and the consequent load sharing ratio of the rock and of the support, must be taken into consideration. Based on practical knowledge and experience, in each case, the designer must determine those variables which are likely to have a major influence on the stability behavior of the supported opening. Then, he must make an assessment on the result which is caused by realistic changes in the variables. The aim is to come up with a balanced design where the strength capacities of the structural elements, i.e., including the rock itself, are fully utilized while the required stability criterion is achieved.

The final stage of the design is to provide the specifications. Excavation method, installation procedures of support elements must be clearly specified. With respect to environmental factors, the allowable influences of nearby excavations must also be specified. Allowable blast effect caused by nearby blasting must be clearly stated. It is essential that the procedures to monitor the performance of the support system, and in some cases to test the structural elements of the system, be specified.

Monitoring and Testing of Support System

Because the design of support systems in mines are largely based on practical knowledge and experience the monitoring and testing of the installed systems is essential and an integral part of the support system design. Stability behavior by a support system different from the one anticipated by the design, indicates faulty design. The sources of error can be numerous, because of the numerous interacting factors. An appropriately designed monitoring scheme is the only method which yield the additional knowledge and experience required for improved design.

The most undesirable effect of a faulty design is the failure of the support system. This can be caused either by the failure of entire system or by the failure of individual structural elements. Whatever the case, failure occurs either because of the acting load is larger than the design load or because of the actual strength is less than the design strength. The latter applies only to the rock, because the ultimate strengths of the man-made structural elements are well defined.

Various monitoring devices are capable of monitoring either load or deformation. Deformation measuring devices are used to indicate possible failure, due to the insufficient strength. The monitoring devices are positioned in such a way that maximum deformation or load increase can be detected. In order to do so, the most likely failure mode of the support system need to be known from experience.

The support system beside being adequately safe, must also be economical. An over-designed support system can also be regarded as a faulty design. For example, the installation cost of any support within a self-supporting opening would be a complete waste. To obtain information on the economic aspect of support design, monitoring is probably the only safe method.

The influences of the environmental factors also need to be monitored. Load and deformation effects are monitored by the previously mentioned devices. Ground vibrations, produced by nearby blasting, are measured by specially designed accelerometers.

All of these measurements are used to improve support design by interpreting the data from the design point of view. They are also used as input data for future design or, if necessary, for redesigning the existing support system.

The other function of monitoring is concerned with safety. The devices can be used to detect deteriorating safety conditions. Some of them are warning devices. They are automatically activated when a predetermined unstable condition is reached.

Testing of support systems involves in situ tests of individual structual elements of the system. The objectives of these tests is to determine the effectiveness of the installed structural element in fulfilling its designed function. The test arrangement, therefore, must closely simulate that function.

ROCK BOLTING

The result of the survey given in Fig. 4 reveals that rock bolting is the most common artificial support system. It is used to various extents in all of the base metal mines in Ontario, regardless of the mining method.

TYPES OF ROCK BOLT

Figure 5 summarizes the various rock bolt types used by the number of mines. Accordingly 39 mines out of 40 are using mechanically anchored bolts. Somewhat more common is the use of forged head bolts than the more expensive bolts threaded at both ends.

Resin grouted rebars are used by 19 mines. In four mines the rebars are tensioned, while in 16 mines they are grouted without tensioning. Fully grouted rebars, using cement grout, are employed by eight mines out of 40.

Cable bolts are used in 12 mines. They are installed with and without tensioning in five and ten mines, respectively. Finally, split set type rock bolts are used in nine mines.

In addition to those listed by Fig. 5 there are several well known types of rock bolts. The present discussion, however, will be restricted to those which are being employed by base metal mines in Ontario.

Mechanically Anchored Bolts

The bolts are anchored by expansion shell anchors of a wide variety, all of which operate in basically the same way. A wedge, attached to the bolt shank, is pulled into the conical anchor shell by turning the shank. As the wedge is pulled, the leaves of the shell are forced to expand against the drillhole walls. At the collar of the hole the bolt tension is transferred to the rock by steel plate. In case of good quality rock masses very high bolt tension, approaching the yield strength of the bolt shank, can be achieved. On the other hand, in poor quality rocks anchor slip takes place due to the local crushing of the rock at the anchor points, at tension below yield. Anchor slippage results in loss of the load carrying capacity and a reduction in the reinforcing capability of the bolt.

The mechanically anchored bolts can be tensioned immediately after installation which is an essential advantage in reinforcing a rock mass. The required reliable anchorage can only be achieved by using correct installation procedure which requires skilled workmen and close supervision.

The particulars of the present mechanically anchored bolting practice of the mining industry in Ontario is given in Fig. 6. The drillhole size ranges between 1 1/8 in. and 1 3/4 in. The most popular size is 1 1/4 in. diameter, used by 34 mines.

The bolt lengths vary from two ft to 14 ft. However, the 6 ft length is far the most common, being used in 35 mines. Next in line is the 8 ft bolt length, followed by 4 ft length, employed by 23 and 14 mines, respectively. The high tensil strength bolt shank diameter is either 5/8 in. or 3/4 in.

Although a few mines are using 5 in. x 5 in. or 6 in. x 6 in. steel plate sizes, the common plate size is 4 in. x 4 in. (used in 37 mines). The plate thickness varies from 3/16 in. to 1/2 in., however, the most popular thicknesses are 1/4 in. and 3/8 in. employed in 23 and nine mines, respectively.

The dominant bolting pattern is 4 ft x 4 ft, used in 29 mines. Other bolt spacings ranging from 2 ft to 5 ft can also be seen in a few mines. In addition, bolts are installed randomly in every mines, to secure rock blocks with critically oriented joints.

The torque for bolt installation is provided in 34 mines by either stopers or jacklegs. In some mines impact wrenches or hand wrenches are employed. Two mines use bolting machines. Bolt installation follows written instructions in 29 mines. Special bolting crews install rock bolts in 11 mines.

In connection with rock bolting the majority of mines are using wire mesh screening, to control loose rocks. The size of employed mesh varies, but the dominant mesh size is 4 in. x 4 in. The diameter of wire mesh varies from 0.0625 in. to 0.3125 in., with No. 9 gauge the most common (0.1483 in. diameter).

Grouted Bolts

If rock quality is suitable, mechanically anchored bolts are preferred because of their lower cost. However, in poor quality rock formations grouted bolts must be used for rock reinforcement. The term of grouting in mining refers to the method of anchorage, i.e., to the manner the load is transmitted between the bolt and the walls of the drillhole. In civil engineering practice mechanically anchored bolts are also grouted, but primarily for corrosion protection of the bolt.

The anchorage element of a resin-grouted bolt is a two-part compound of resin and its catalyst, usually with an inert filler to reduce the cost. The hole is drilled to the required depth and the cartridge containing the separated resin and catalyst is pushed in the hole by the bolt. The bolt is forced into the cartridge and rotated for a few seconds to mix the resin and catalyst. Cartridges of various resins are available with various setting time.

The resin bolt has the advantage that it can be used where expansion shell bolts cannot be used due to either weak rock or irregular diameter holes. They are more expensive than mechanically anchored bolts. To reduce the cost, rebars are often used instead of regular resin bolts.

In weak or heavily fractured rocks the resin-grouted rebars are usually grouted for their full length. The fragmented rock is held along the walls of the drillhole by the set resin which is reinforced by the rebar. Steel plates are seldom used in this type of application.

Steel plates are used when the installation involves bolt tensioning. In this case the rebar is anchored in the hole by the resin grout which sets within minutes. Usually 30 minutes after installation the rebar, having a threaded end, can be tensioned.

In some cases, resin grouted rebars are both stressed and grouted for their full length. The rebars, threaded at one end, are installed by using two different types of cartridges, inserted simultaneously into the drillhole. After the fast-setting resin at the anchorage point reaches a specified strength, the rebar can be stressed, while the slow-setting resin along the bar is still in its liquid state.

Another type of grouted bolt, used in some mines, is a rebar with cement grout. A thick grout is pumped into the hole by means of a simple hand pump or a mono-pump. The rebar is then pushed into the grout. In up-holes the bar, until the grout is set, must be secured against slippage by wooden or steel wedge inserted at the collar of the hole.

The particulars of the present grouted bolting practice used in the mines of Ontario is given in Fig. 7. The drillhole diameter ranges between 1 1/8 in. and 2 in. The most common size is 1 1/4 in. used by 17 mines. The diameter of rebar used varies between 3/4 in. and 1 in.; 7/8 in. diameter is the most popular diameter.

There are large variations in the rebar length used; they range from 3 ft to 20 ft. Up to 8 ft length, rebars are mainly resin grouted, while rebars with lengths over 8 ft are commonly cement-grouted.

If a steel plate is used the most common size is 4 in. by 4 in. However, almost an equal number of mines install rebars without face plates.

The dominant bolting pattern is 4 ft x 4 ft. Spacings of 3 ft and 6 ft, however, are also used by some mines. In addition to regular bolting patterns, random rebars are also used for securing critical rock blocks.

The majority of mines which employ grouted bolts follow written installation procedures. In about one-third of the mines, bolts are installed by special bolting crews.

The use of wire mesh is less common in the case of grouted rebars than in the case of mechanically anchored rock bolts. The size of mesh used in six mines is 4 in. x 4 in. with wire diamters of 0.1483 in. and 0.1920 in., i.e., No. 9 and 6 gauge, respectively.

Cable Bolts

In conjunction with long drillholes the frequently used bolt type is the cable bolt.

The maximum reinforcing effect of a cable bolt is obtained when it is tensioned. Wire rope, usually of several strands, is first anchored in the hole by either cement grout or special anchor shell. The cement grout anchor, however, is more common. After a few days to cure the cement, the cable is tensioned with pulling jacks and then the rope is locked in its tensioned state. Mainly for corrosion protecion, the entire length of drillhole is then grouted. The technique is expensive and is limited to major operations.

Cable bolts are also installed without tensioning. The reinforcement of the fractured rock mass is provided along the walls of the drillhole by the set cement grout which is in turn reinforced by the cable.

As indicated in Fig. 5, out of 40 Ontario's mines 12 mines use cable bolts. Non-tensioned and tensioned cable bolts are installed in ten and five mines, respectively.

Split Sets

This type of rock bolt is a 1 1/2 in. diameter split tube of 0.09 in. wall thickness, with a face plate secured to one end of the tube. As the

vent the critical surface rock blocks from falling out which in turn prevents the neighbouring blocks moving. This is an analog to the role of the "keystone" in the stability of a masonary arch. As well, the case of tensioned bolts cracks and joints are kept close which is considered an additional surface effect, i.e., decreasing the chance of deterioration by air and water attack. This theory of support mechanism is widely used in bolting practice, when the random bolts are installed.

Design Procedures

The laboratory and field investigations, conducted by the various research workers, had been based on the foregoing theories of operative support mechanisms with rock bolts. Based on the obtained data various design procedures have been formulated. The following procedures are judged to be the most advances and most accepted in bolting practice.

Rock-support interaction analysis method

The comprehensive rock bolting design procedure, based on rock-support interaction analysis, was developed by Hoek and Brown (7). The analysis embraces all the essential factors of underground support design. However, due to the large number of factors which have to be considered, the procedure is based on the following simplifying assumptions:

(a) Two-dimensional circular opening;
(b) Equal horizontal and vertical in situ stresses;
(c) Uniform radial support pressure on the walls of the opening;
(d) Linear-elastic material properties for the original rock mass;
(e) Perfectly plastic material properties for the broken rock mass;
(f) The volumetric strains are governed in the elastic and the plastic regions by the elastic constants and the flow rule, respectively;
(g) Neither the original nor the broken rock masses exhibit time-dependent behaviour;
(h) The radius of the plastic zone depends upon the in situ stress, the support pressure and the material characteristics of both the original and broken rock mass;
(i) Radial symmetry about the axis of the opening.

The analysis requires the following input data:

σ_c = uniaxial compression strength of intact rock pieces;
m,s = material constant for original rock mass;
E = modulus of elasticity of original rock mass;
ν = Poisson's ratio of original rock mass;
m_r, s_r = material constant for broken rock mass;
γ_r = unit weight of broken rock mass;
p_o = in situ stress magnitude;
r_i = radius of opening;
l = free bolt or cable length;
d_b = bolt diameter or equivalent cable diameter;
E_b = elastic modulus of bolt or cable material;
Q = load-deformation constant for anchor and head;
T_{bf} = ultimate failure load from pull-out test;
s_c = circumferential bolt spacing;
s_l = longitudinal bolt spacing.

The rock-support interaction analysis design procedure is demonstrated in Fig. 8, where the radial support pressure is plotted against the radial deformation of the circular opening.

Point A in Fig. 8 represents the pressure condition prior to excavation. The rock mass is in equilibrium, therefore, the internal pressure p_i acting across the future profile of opening is equal to the in situ stress p_o.

Due to the excavation the support pressure p_i, previously provided by the rock inside the opening, drops to zero. However, the roof remains stable because the radial deformation u is restrained by the proximity of face. Let us assume, for example, that the internal support pressure provided by the face is indicated by point B on the load-deformation curve of the roof. It should be recognized that the roof, sidewalls and floor are characterized by three different load-deformation curves as shown in Fig. 8. Because of the weight of the loosened rock the support pressure required to limit the stress-induced deformation is highest in the roof.

Let us further assume that the roof is rock bolted at this stage of face advancement. Other than the installation tension as shown by point D the bolts carry no load.

As the face advances the restraint provided by the face reduces considerably. The result is an increased radial deformation of the roof (also of sidewalls and floor) indicated by the position of load-deformation curve beyond point B. The radial deformation of roof induces load in the bolts. The support pressure available from the bolts increases with the radial deformation, as shown in Fig. 8 by the line DC.

The restraint provided by the face against radial deformation with the further face advancement becomes less and less until it completely diminishes. If no bolting had been installed with face advance the radial deformation of the roof would increase as indicated by the broken line beyond point C. However, because the support pressure available from the installed rock bolts and the support pressure required to limit further roof deformation are balanced at point C, a state of equilibrium is created. That is, theoretically and for the assumed conditions of the analysis.

The load-deformation curves of the sidewalls and floor intercepts the radial deformation axis at points E and F, respectively. Because at these radial deformations, represented by these points, the pressure required to limit further deformation are zero, the sidewalls and floor are stable. In Fig. 8 rock bolting is indicated for the roof only. If bolts were installed in the sidewalls they would limit their deformation to the value represented by the intersection point G of the load-deformation curves of the sidewalls and the bolts.

The well defined steps of the design procedure are given by Hoek and Brown (7). The design method is limited to end-anchored bolts. As stated by the authors the rock-support interaction concept cannot be applied to the bolt types which are grouted for their full length with the purpose of reinforcing the rock mass by binding the fragmented rock along the walls of the drillhole. "This is because they do not act independently of rock mass and hence the deformations which occur in both rock mass and support system cannot be separated".

One of the prime objectives of an ongoing investigations in Japan is to establish design procedure for fully bonded bolts. The results of theoretical analyses, laboratory tests, finite element models and field deformation measurements were successfully correlated by Tanimoto et al. (8,9,10) to establish design procedure applicable to their specific case of investigation. Tunnels in soft, fractured and weathered rocks were successfully bolted with

fully bonded bolts by using the developed design procedure. Although the suggested procedure is rather complex, it represents a sound approach to solve the difficult problem of interdependent deformation of rock mass and fully bonded rock bolts. However, the procedure will have to be tested for various field and ground conditions before it can be adopted as a rock bolt design practice. Field investigations are required to measure the deformation of the rock mass supported by fully bonded rock bolts and the deformation of the bolts themselves. The available support pressure rendered by the bolts, bonded within various rocks in various in situ conditions, is the other required design parameter which can only be established by field investigations.

Code of practice method

This method recommends the design of rock bolting systems by following the rules of practice based on the results of studies conducted in mines and laboratories. The formulated rules describes the procedures to be followed in selecting rock bolts, in designing patterns, and in monitoring various aspects of ground reaction, that are applicable to the majority of the cases. The scope of the method is restricted to the rock bolt reinforcement of areas of roofs or backs for a limited period of time common in construction before permanent linings are placed and in mining stopes and drifts within mining blocks. It does not cover the use of bolts for such purposes as in drawpoints, walls of shafts, and permanent installations. It deals only wiht the conventional end-anchored tensioned rock bolts installed in a selected site in a systematic pattern. Special conditions, such as faults require special consideration, and are not covered by the method.

The two codes frequently used, were developed by CANMET (6) and by the Australian Snowy Mountains Hydroelectric Authority (5). A brief description of the CANMET's code, which is the most popular, is given below:

The bolt assemblage is selected explicitly for the rock. Methods of rock classification, rock testing and pull testing are specified. The anchorage capacity of the bolt/rock system is established in the following terms:

Q_y = representative yield load of system,
Q_m = maximum yield load of system
Q_r = representative load capacity of the anchorage.

The representative load capacity of the anchorage, Q_r, must be equal to Q_y or 0.7 Q_m whichever is the lesser.

The load capacity of the rock bolt, Q_a, must be equal to either the load capacity of the steel, Q_s, or the load capacity of the anchorage, Q_r, whichever is less.

Bearing plates should conform to standard strength tests. The standard plate is 15 cm square. If the collar of the hole is consistently broken and exceeds 10 cm then the bearing plate must be increased to a square with a side dimension of (D + 5) cm, where D is a representative collar size not exceeded by more than 10 per cent of the holes. Plates other than square must have an equivalent bending strength.

The thickness of the plate is also adjusted to suit hole collar size. If t_4 is a thickness of plate found to be suitable for a 10 cm collar, then the desired thickness t is derived from:

$$\frac{t}{t_4} \geq \frac{D}{D_4}^{4/3}$$

where D is the width of the plate and D_4 is 10 cm.

The minimum length of bolt L should be either 1.2 m to avoid fractured ground, or more than the depth of a rock block containing the collar of the hole. The maximum length of bolt, if not governed by the dimension of the opening, is determined from:

$$L \leq Q_a/s^2g$$

where s is the average spacing of the bolts and g is the average density of the rock involved. If the bolt cannot be installed with a tensile load equal to or greater than 0.5 Q_a, then the maximum length should be obtained from:

$$L \leq Q_a/2s^2g$$

The bolt spacing, s, should be three times the representative spacing of the joints, cross-bed fractures, etc., unless a membrane (e.g. lagging,

wiremesh, gunite or shotcrete) is provided, in which case the maximum spacing would depend on the type of membrane. The maximum spacing in any case is to be limited to 0.9 L if L is greater than one quarter of the breadth or span of the opening, or 0.5 L if L is less than one quarter of the opening. At an intersection the span means the diagonal distance between abutments.

The bolts should normally be tensioned to a load of between 0.5 Q_a and 0.8 Q_a. If tightened by torque action then the maximum torque, T, should be calculated from:

$$T = 0.8\, r^3 \sigma_y$$

where r is the radius of the bolt at maximum section and σ_y is the specified minimum yield strength of the steel in tension.

In yielding rock, or where further excavation is to take place the bolts should be periodically checked with a torque device or dynamometer, or by pull tests with a jack. If 10 per cent of the bolts sampled are shown to have a tension less than 0.5 Q_a, either the slack bolts must be retensioned or extra bolts added.

Monitoring may also be continuously carried out by borehole extensometers and closure measurement devices.

POST-FAILURE TESTING

Investigation of heavily loaded rock mass structures indicates that regions of un-failed and failed rock coexist simultaneously in a rock mass. To carry out stability analysis on such structures, therefore, an adequate knowledge of rock behaviour is necessary, not only up to failure but also beyond failure.

To provide adequate rock properties data for stability analyses of underground openings and support systems, taking into account rock post-failure behaviour, an up-dating of the test capabilities of the Rock Mechanics Laboratory become essential.

The various candidate test equipment described in the literature were reviewed and analyzed for this purpose. The offered advantages and disadvantages, the involved costs, the required development time and the complexity

of the various candidate systems were evaluated and compared as part of the selection process.

The evaluation resulted in the adaption of a particular equipment system and development of testing procedures which will meet analytic requirements concerning post-failure rock properties (11).

CONCLUSIONS AND RECOMMENDATIONS

1. The completed study revealed that the existing information and knowledge are sufficient to define those factors which generally have major influence on the stability of underground openings and consequently on support design. One must, however, be aware that the relationships between the variables probably never could be expressed by exact mathematical equations. Therefore, the design of a support system for an underground opening in rock will remain an art involving the application of practical knowledge and experience. Investigations involving laboratory testing, field measurements and analytical models are essential. The obtained data, translated into understanding of the interplay between the influencing variables, will then be used for the containuous improvement of the art of support system design.
2. The rock-support interacion analysis method is suitable for rock bolting design. It is limited to end-anchored, both mechanical or grout type anchors, bolts. The design procedure is tailored to civil engineering design practice which might represent a draw-back in gaining overall acceptance in mining engineering practice.
3. More familiar to the mining engineering design approach is the code of practice method. The given procedure is again limited to end-anchored bolts. Recommended procedures are based on the results of studies which were conducted prior to 1970, as well as, on the understanding of relationships between the influencing variables as it prevailed at that time.
4. The more recent rock-support interaction analysis method, beside including up-to-date measurement data, is based on the present and more advanced state of understanding and on the results of the latest technological and analytical developments.

5. It is desirable to combine the above advantages in one package. The planned rock bolting guidelines would, therefore, follow the design approach set by the code of practice method and would be improved by latest data base and development which are contained in the rock-support interaction analysis method.
6. On this basis a rock bolting guideline is to be drafted during the next phase of the Material Properties and Support Systems Project. First an adequate number of rock bolting support systems used in Ontario's mines are to be checked by both design methods. Then, by considering the prevailing factors in each case, the results are to be analyzed and correlated. The recommendations and rules of the draft guidelines are then to be formulated according to the findings of the investigations.
7. The design procedure to secure rock blocks with critcally oriented joints is well defined. No further investigation is required.
8. The existing data with respect to interdependent deformation of rock mass and fully bonded rock bolts as well as data on the support pressure provided by the bonded bolts to the rock mass are insufficint for the development of design procedures. Essential data can only be obtained through systematic field measurement programs.
9. The post-failure strength and deformation properties are essential input for any support system design. The initial tests, conducted with the newly developed equipment, provide the assurance that this requirement can be fulfilled.

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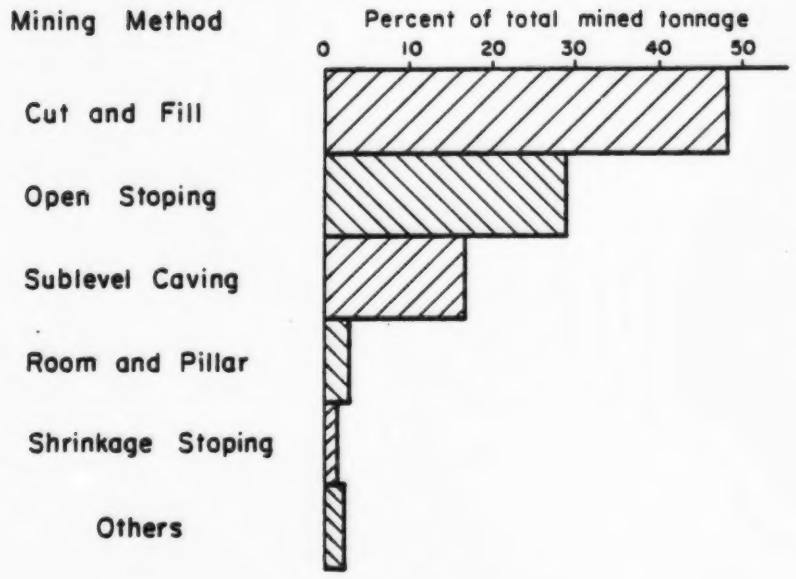

Fig. 1 - Mining methods and relation productions in Canadian base metal mines

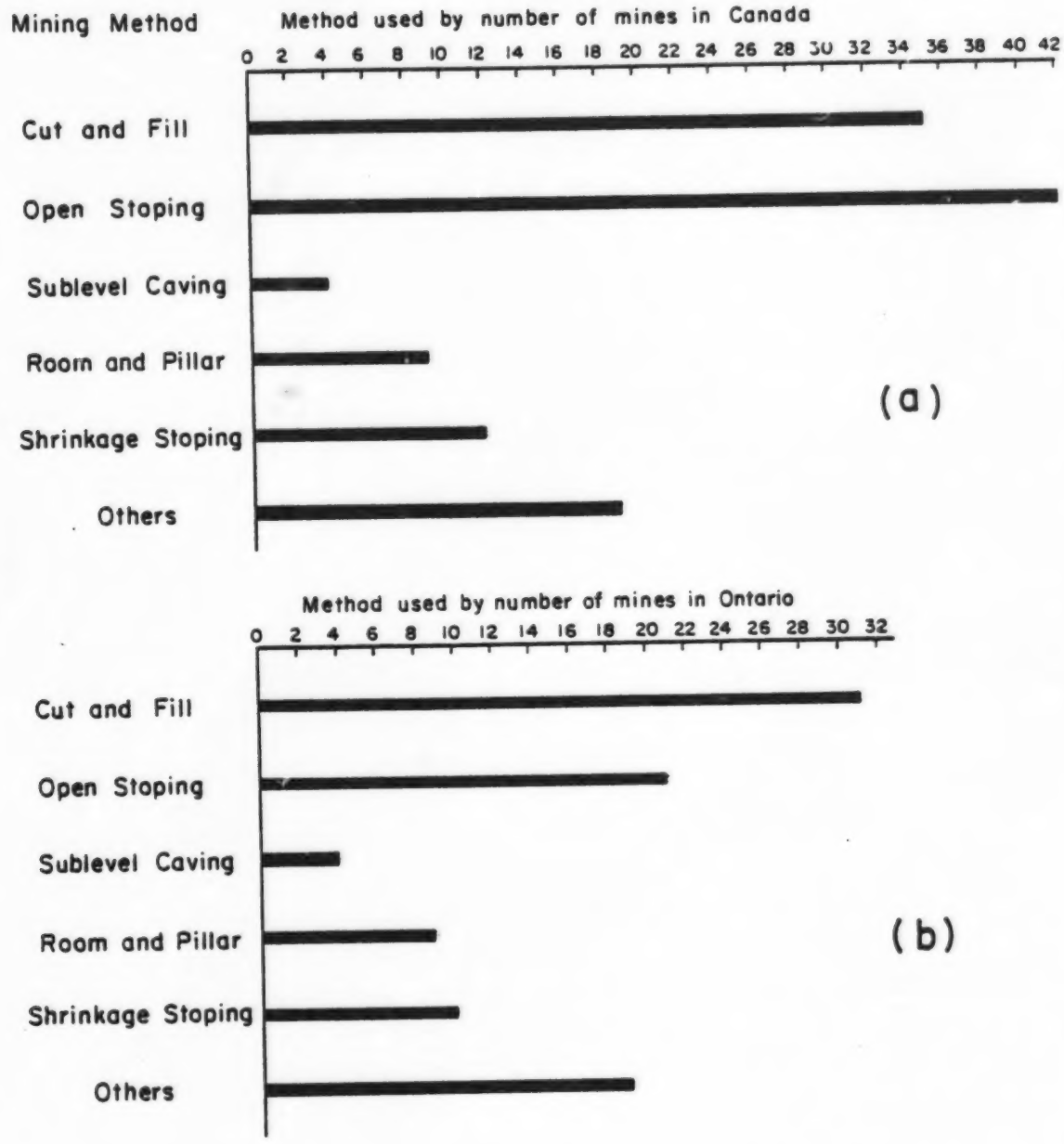
Mining Method
Method used by number of mines in Canada
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40 42
Cut and Fill
Open Stoping
Sublevel Caving
Room and Pillar
Shrinkage Stoping
Others
(a)
Method used by number of mines in Ontario
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32
Cut and Fill
Open Stoping
Sublevel Caving
Room and Pillar
Shrinkage Stoping
Others
(b)

Fig. 2 - Comparison of mining methods used in base metal mines; (a) Canada, (b) Ontario

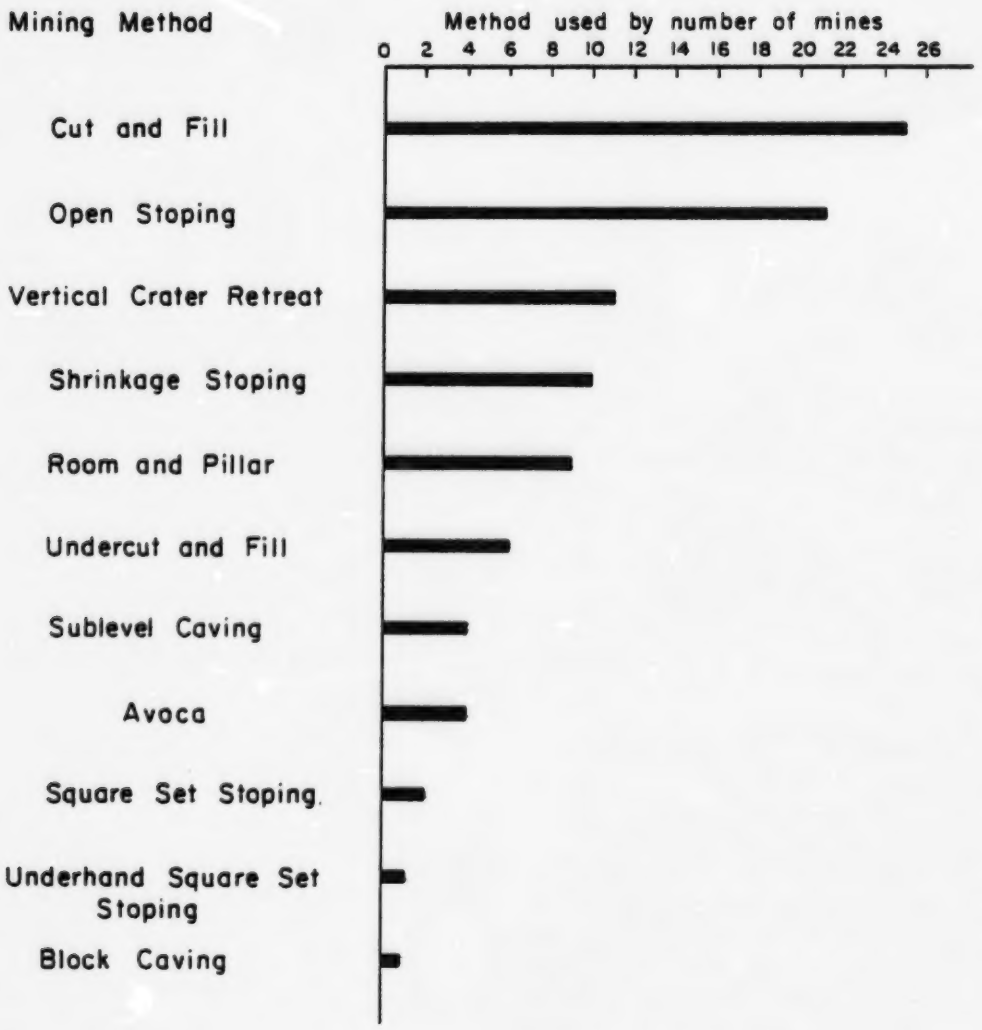

Fig. 3 - Mining methods used by Ontario base metal mines

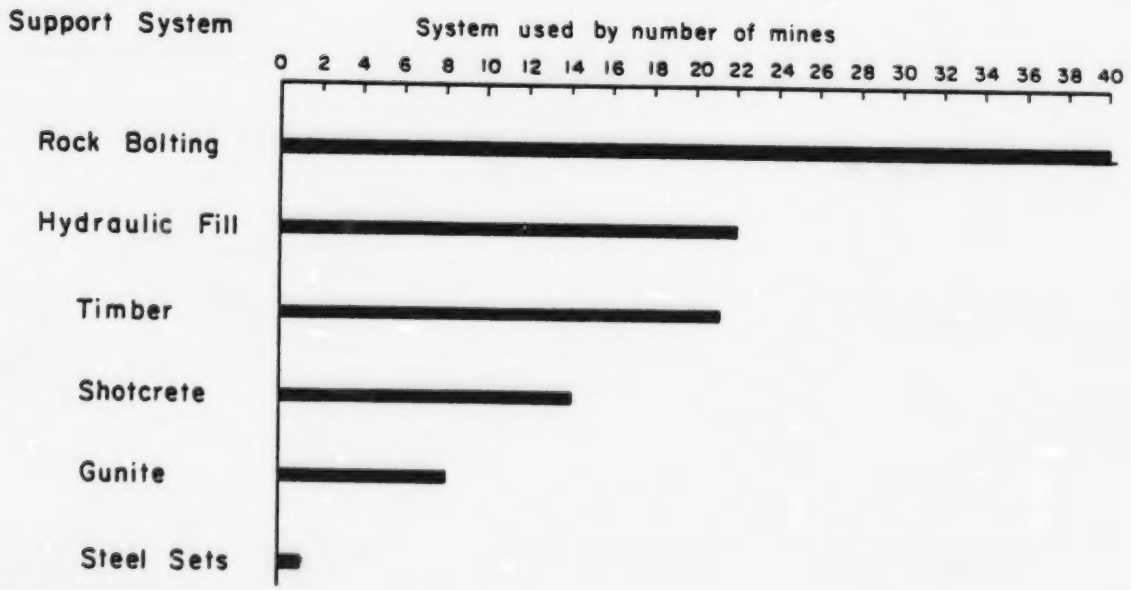
Support System
System used by number of mines
0 2 4 6 8 10 12 14 16 18 20 22 24 26 28 30 32 34 36 38 40
Rock Bolting
Hydraulic Fill
Timber
Shotcrete
Gunite
Steel Sets

Fig. 4 - Support systems used by Ontario base metal mines

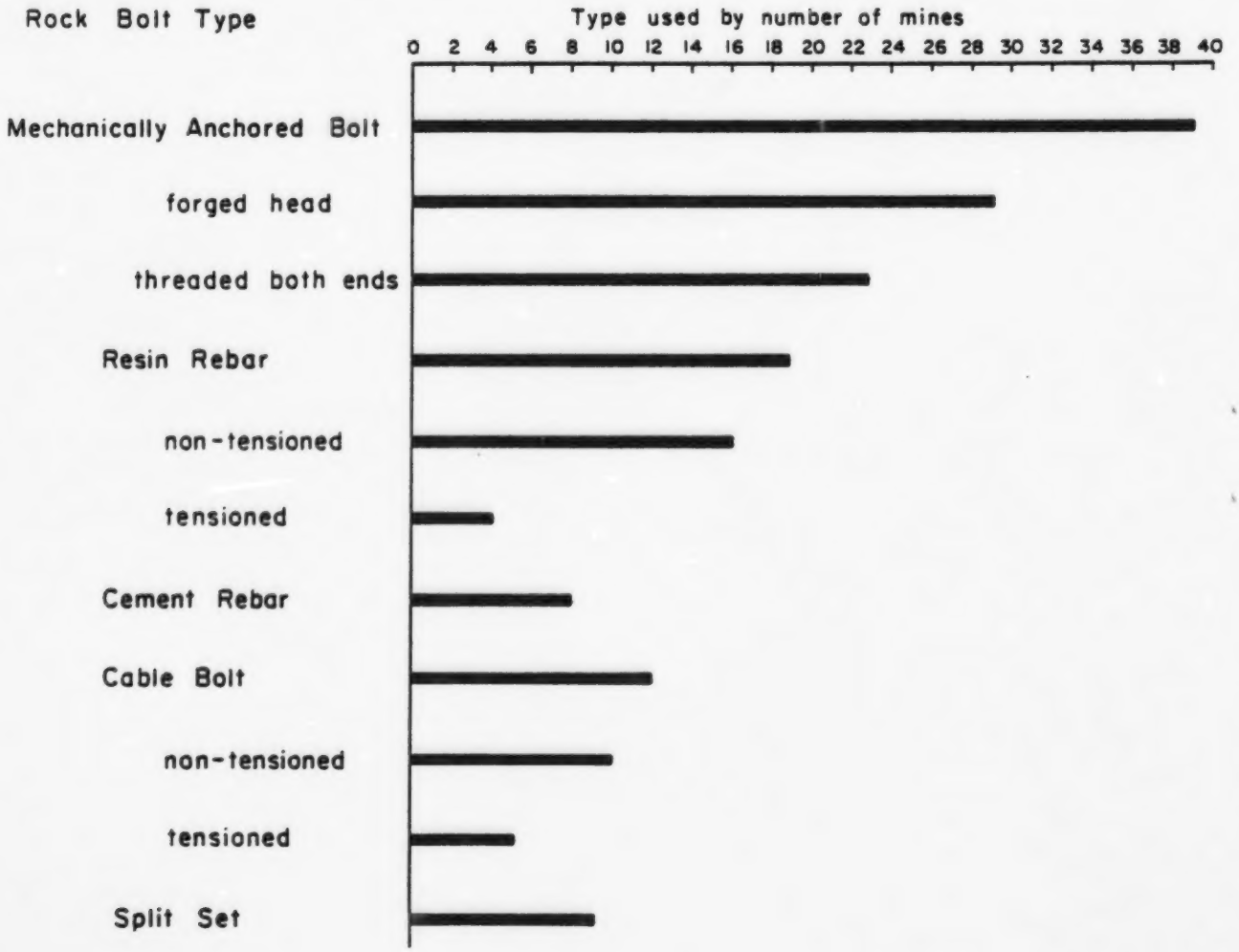

Fig. 5 - Rock bolt types used by Ontario base metal mines

Fig. 6 - Mechanically anchored bolting practice in Ontario base metal mines

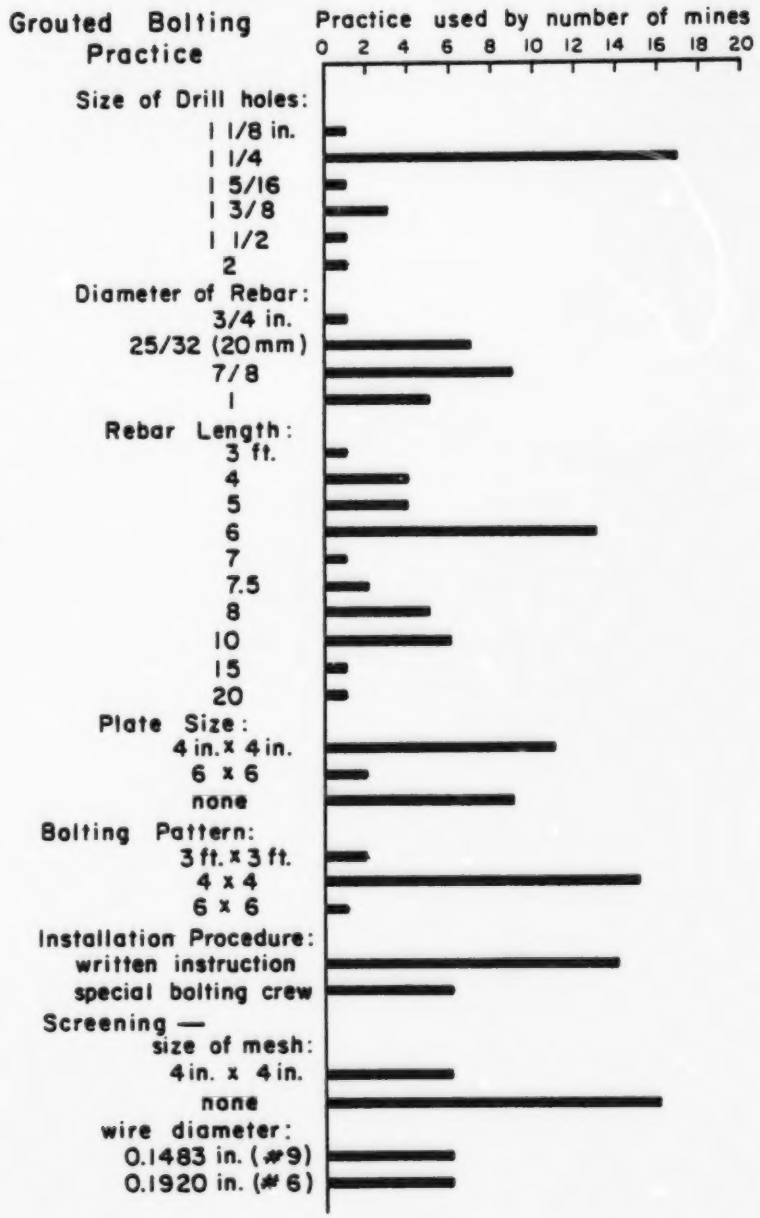

Fig. 7 - Grouted bolting practice in Ontario base metal mines

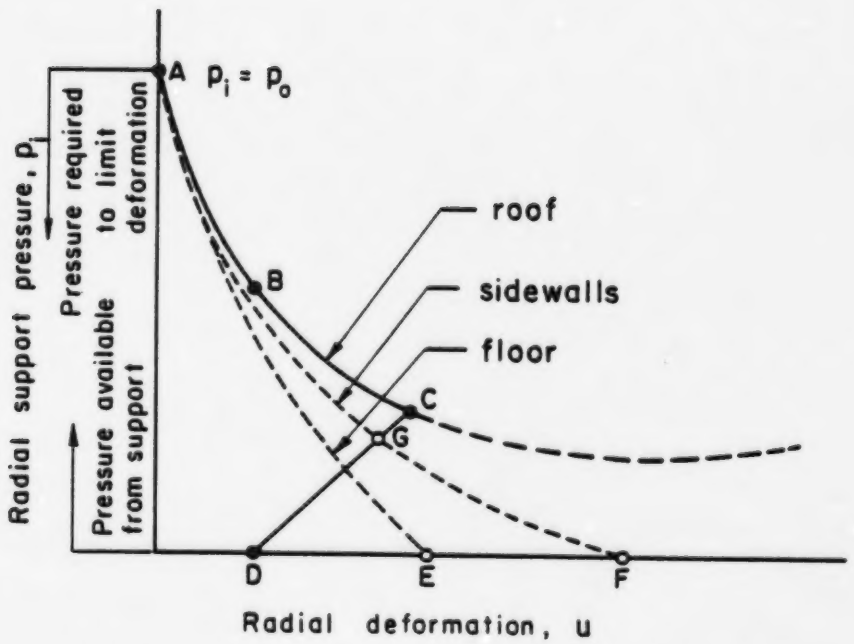

Fig. 8 - Principle of rock-support interaction analysis